

Section 3 GENERAL REQUIREMENTS

3.1 APPLICABILITY

The provisions herein shall apply to bridges of conventional slab, beam girder, box girder, and truss superstructure construction. For other types of construction (i.e. cable stayed and suspension), the Owner shall specify and/or approve appropriate provisions. Seismic effects for box culverts and buried structures need not be considered, except where they cross active faults. The potential for soil liquefaction and slope movements shall be considered.

3.2 SEISMIC PERFORMANCE OBJECTIVES

Bridges shall be designed for the life safety performance objective of Table 3.2.1. Higher levels of performance may be used with the authorization of the bridge owner. Development of design earthquake ground motions for the probabilities of exceedance in Table 3.2-1 are given in Article 3.4.

When required by the provisions of this specification, seismic performance shall be assured by verifying that displacements are limited to satisfy geometric, structural and foundation constraints on performance.

Table 3.2-1 Design Earthquakes and Seismic Performance Objectives

		Performance Objective ⁽¹⁾
Probability of Exceedance (PE) For Design Earthquake Ground Motions ⁽⁴⁾		Life Safety
Maximum Considered Earthquake (MCE) 3% PE in 75 years/or 1.5 Median Deterministic	Service ⁽²⁾	Significant Disruption
	Damage ⁽³⁾	Significant
Expected Earthquake (EE) 50% PE in 75 years	Service	Immediate
	Damage	Minimal

Notes:

(1) Performance Objectives

These are defined in terms of their anticipated performance objectives in the upper level earthquake. Life safety in the MCE event means that the bridge should not collapse but partial or complete replacement may be required. Since a dual level design is required the Life Safety performance level will have immediate service and minimal damage for the expected design earthquake.

(2) Service Levels*:

- *Immediate* – Full access to normal traffic shall be available following an inspection of the bridge.
- *Significant Disruption* – Limited access (Reduced lanes, light emergency traffic) may be possible after shoring, however the bridge may need to be replaced.

(3) Damage Levels*:

- Minimal – Some visible signs of damage. Minor inelastic response may occur, but post-earthquake damage is limited to narrow flexural cracking in concrete and the onset of yielding in steel. Permanent deformations are not apparent, and any repairs could be made under non-emergency conditions with the exception of superstructure joints.
- Significant – Although there is no collapse, permanent offsets may occur and damage consisting of cracking, reinforcement yield, and major spalling of concrete and extensive yielding and local buckling of steel columns, global and local buckling of steel braces, and cracking in the bridge deck slab at shear studs on the seismic load path is possible. These conditions may require closure to repair the damage. Partial or complete replacement of columns may be required in some cases. For sites with lateral flow due to liquefaction, significant inelastic deformation is permitted in the piles, whereas for all other sites the foundations are capacity-protected and no damage is anticipated. Partial or complete replacement of the columns and piles may be necessary if significant lateral flow occurs. If replacement of columns or other components is to be avoided, the design approaches producing minimal or moderate damage (Figure C3.3-1) such as seismic isolation or the control and reparability design concept should be assessed.

* See commentary and design sections for geometric and structural constraints on displacements and deformations.

(4) The upper-level earthquake considered in these provisions is designated the Maximum Considered Earthquake, or MCE. In general the ground motions on national MCE ground motion maps have a probability of exceedance (PE) of approximately 3% PE in 75 years. However, adjacent to highly active faults, ground motions on MCE maps are bounded deterministically as described in the commentary for Article 3.2. When bounded deterministically, MCE ground motions are lower than ground motions having 3% PE in 75 years. The performance objective for the expected earthquake is either explicitly included as an essentially elastic design for the 50% PE in 75 year force level or results implicitly from design for the 3% PE in 75 year force level.

3.3 SEISMIC DESIGN APPROACH

All bridges and their foundations shall have a clearly identifiable earthquake resisting system (ERS) selected to achieve the performance objectives defined in Table 3.2-1. The ERS shall provide a reliable and uninterrupted load path for transmitting seismically induced forces into the ground and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements. All structural and foundation elements of the bridge shall be capable of achieving anticipated displacements consistent with the requirements of the chosen mechanism of seismic resistance and other structural requirements.

3.3.1 Earthquake Resisting Systems (ERS)

For the purposes of encouraging the use of appropriate systems and of ensuring due consideration of performance by the owner, the ERS and earthquake resisting elements (ERE) are categorized as follows:

- Permissible
- Permissible with Owner Approval
- Not Recommended for New Bridges

These terms apply to both systems and elements. For a system to be in the permissible category, its primary ERE must all be in the permissible category. If any ERE are not permissible, then the entire system is not permissible.

Permissible systems (Figure C3.3.1-1a and -1b) and elements have the following characteristics:

1. All significant inelastic action shall be ductile and occur in locations with adequate access for inspection and repair. Piles subjected to lateral movement from lateral flow resulting from liquefaction are permitted to hinge below the ground line with the owners' approval. If all structural elements of a bridge are designed elastically ($R=1.0$ and Article 4.10) then no inelastic deformation is anticipated and elastic

elements are permissible, but ductile detailing is required.

2. Inelastic action of a structural member does not jeopardize the gravity load support capability of the structure (e.g. cap beam and superstructure hinging)

Permissible systems that require owner approval (Figure C3.3.1-2) are those systems that do not meet either item (1) or (2), above. Such systems may only be used with the owners' approval. Additionally, these systems will require the use of the highest level of analysis (Seismic Design and Analysis Procedures E – SDAP E), as outlined in the flow chart shown in Figure 3.3.1-1. The minimum Seismic Design and Analysis Procedures (SDAP) are defined in Article 3.7.

In general, systems that do not fall in either of the two permissible categories (Figure C3.3.1-3) are not allowed. However, if adequate consideration is given to all potential modes of behavior and potential undesirable failure

mechanisms are suppressed, then such systems may be used with the owner's approval.

The interrelationship between the performance objective and the ERS is given in Table 3.3.1-1. Abutment design issues are amplified in Table 3.3.1-2.

3.4 DESIGN GROUND MOTION

Design response spectra acceleration parameters shall be obtained using either a general procedure (Article 3.4.1) or a site-specific procedure (Article 3.4.3). A site-specific procedure shall be used if any of the following apply:

- Soils at the site require site-specific evaluation (i.e. Site Class F soils, Article 3.4.2.1), unless a determination is made that the presence of such soils would not result in a significantly higher response of the bridge.

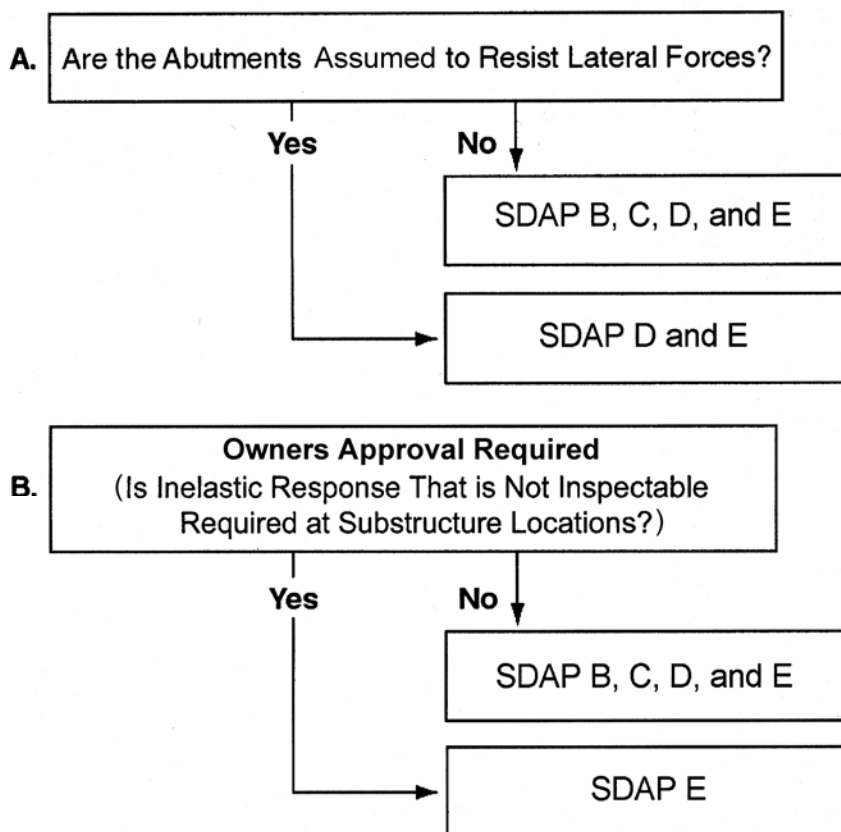


Figure 3.3.1-1 Classification of ERS

- The bridge is considered to be a major or very important structure for which a higher degree of confidence of meeting the seismic performance objectives of Article 3.2 is desired.
- The site is located within 10 km (6.25 miles) of a known active fault and its response could be significantly and adversely influenced by near-fault ground motion characteristics.

3.4.1 Design Spectra Based on General Procedure

Design response spectra for the rare earthquake (MCE) and expected earthquake shall be constructed using the accelerations from national ground motion maps described in this section and site factors described in Article 3.4.2. The construction of the response spectra shall follow the procedures described below and illustrated in Figure 3.4.1-1.

Design earthquake response spectral acceleration at short periods, S_{DS} , and at 1 second period, S_{D1} , shall be determined from Eq. 3.4.1-1 and 3.4.1-2, respectively:

$$S_{DS} = F_a S_s \quad (3.4.1-1)$$

and

$$S_{D1} = F_v S_1 \quad (3.4.1-2)$$

where S_s and S_1 are the 0.2-second period spectral acceleration and 1-second period spectral acceleration, respectively, on Class B rock from ground motion maps described below and F_a and F_v are site coefficients described in Article 3.4.2.3. Values of S_s and S_1 may be obtained by the following methods and need not exceed the values of (c):

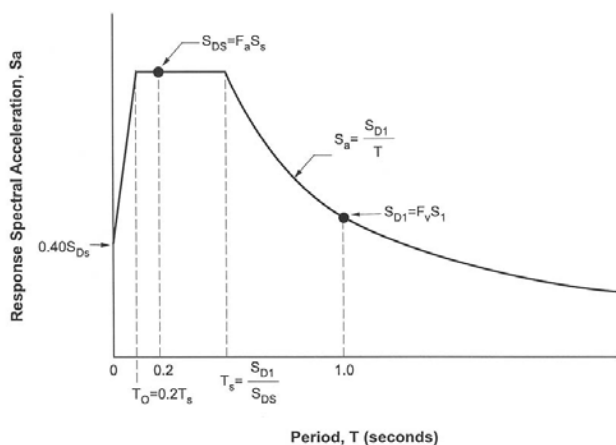
1. For the MCE
 - (a) S_s and S_1 may be obtained from national ground motion maps (Figures 3.4.1-2(a) through 3.4.1-2(l)).
 - (b) S_s and S_1 may be obtained from the CD-ROM published by the U.S. Geological Survey (Leyendecker et al., 2000a) for site coordinates specified by latitude and longitude, or alternatively, by zip code.
 - (c) S_s need not exceed 1.5g and S_1 need not exceed 0.6g. If these upper bound limits apply, then SDAP E of Article 4.6 shall be used for design and the 1.5 multiplier in Equations 7.3.5-1 and 8.3.5-1 shall be replaced with the greater of 1.5, $0.8S_s$, or $2S_1$, but in no case shall the multiplier exceed 2.0.
2. For the expected earthquake, S_s and S_1 may be obtained from national ground motion maps (Figures 3.4.1-3(a) through 3.4.1-3(d)).

Table 3.3.1-1 Performance Objectives and Earthquake Resisting Systems

Performance Objectives	Expected Element Behavior	Earthquake Resisting System	Abutment Performance	
			50% in 75 Years	3% in 75 Years
Life Safety	Linear Elastic Nonlinear Elastic Nonlinear Inelastic	Permissible elements designed to resist all seismic loads within displacement constraints. Elements requiring owner approval are OK.	Limited damage and soil passive mobilization O.K.	Significant damage. Soil passive mobilization is O.K.

Table 3.3.1-2 Abutment Design Issues

Significant Damage Accepted	
ERS does not Include Abutment Contribution	ERS Includes Abutment Contribution
The ERS is designed to resist all seismic loads without any contribution from abutments (SDAP B and C). Abutments then limit displacement and provide additional capacity and better performance. The bridge is safe even if serious problems occur at the abutments. For SDAP D and E and the 50% in 75-year event, the bridge should be analyzed with the abutments and the abutments are designed for the 50% in 75-year forces and displacements. If sacrificial concrete shear keys are used to protect the piles, the bridge shall be analysed with all combinations of shear key failure considered (i.e. at each abutment separately and both abutments simultaneously).	The ERS is designed with the abutments as a key element of the ERS. Abutment are designed and analyzed for the 3% in 75-year forces and displacements.

**Figure 3.4.1-1 Design Response Spectrum, Construction Using Two-Point Method**

The design response spectrum curve shall be developed as indicated in Figure 3.4.1-1 and as follows:

1. For periods less than or equal to T_0 , the design response spectral acceleration, S_a , shall be defined by Equation 3.4.1-3:

$$S_a = 0.60 \frac{S_{DS}}{T_0} T + 0.40 S_{DS} \quad (3.4.1-3)$$

T and T_0 are defined in 2 below.

Note that for $T=0$ seconds, the resulting value of S_a is equal to peak ground acceleration, PGA.

2. For periods greater than or equal to T_0 and less than or equal to T_s , the design response spectral acceleration, S_a , shall be defined by Equation 3.4.1-4:

$$S_a = S_{DS} \quad (3.4.1-4)$$

where $T_0 = 0.2T_s$, and $T_s = S_{D1}/S_{DS}$, and T = period of vibration (sec).

3. For periods greater than T_s , the design response spectral acceleration, S_a , shall be defined by Equation 3.4.1-5:

$$S_a = \frac{S_{D1}}{T} \quad (3.4.1-5)$$

Response spectra constructed using maps and procedures described in Article 3.4.1 are for a damping ratio of 5%.

3.4.2 Site Effects on Ground Motions

The generalized site classes and site factors described in this section shall be used with the general procedure for constructing response spectra described in Article 3.4.1. Site-specific analysis of soil response effects shall be conducted

where required by Article 3.4 and in accordance with the requirements in Article 3.4.3.

3.4.2.1 Site Class Definitions

The site shall be classified as one of the following classes according to the average shear wave velocity, SPT blow count (N-value), or undrained shear strength in the upper 30 m (100 ft) of site profile. Procedures given in Article 3.4.2.2 shall be used to determine the average condition.

- A Hard rock with measured shear wave velocity, $\bar{V}_s > 1500$ m/s (5000 ft/sec)
- B Rock with $760 \text{ m/s} < \bar{V}_s \leq 1500 \text{ m/s}$ ($2500 \text{ ft/sec} < \bar{V}_s \leq 5000 \text{ ft/sec}$)
- C Very dense soil and soft rock with $360 \text{ m/s} < \bar{V}_s \leq 760 \text{ m/s}$ ($1200 \text{ ft/sec} < \bar{V}_s \leq 2500 \text{ ft/sec}$) or with either $\bar{N} > 50$ blows/0.30 m (blows/ft) or $\bar{S}_u > 100$ kPa (2000 psf)
- D Stiff soil with $180 \text{ m/s} \leq \bar{V}_s \leq 360 \text{ m/s}$ ($600 \text{ ft/sec} \leq \bar{V}_s \leq 1200 \text{ ft/sec}$) or with either $15 \leq \bar{N} \leq 50$ blows/0.30 m (blows/ft) or $50 \text{ kPa} \leq \bar{S}_u \leq 100 \text{ kPa}$ ($1000 \text{ psf} \leq \bar{S}_u \leq 2000 \text{ psf}$)
- E A soil profile with $\bar{V}_s < 180 \text{ m/s}$ (600 ft/sec) or with either $\bar{N} < 15$ blows/0.30 m (blows/ft) or $\bar{S}_u < 50 \text{ kPa}$ (1000 psf), or any profile with more than 3 m (10 ft) of soft clay defined as soil with $PI > 20$, $w \geq 40$ percent, and $\bar{S}_u < 25$ kPa (500 psf)

Table 3.4.2-1 Site Classification

Site Class	\bar{V}_s	\bar{N} or \bar{N}_{ch}	\bar{S}_u
A	$> 1500 \text{ m/sec}$ ($> 5000 \text{ ft/sec}$)	—	—
B	$760 \text{ to } 1500 \text{ m/sec}$ ($2500 \text{ to } 5000 \text{ ft/sec}$)	—	—
C	$360 \text{ to } 760 \text{ m/sec}$ ($1200 \text{ to } 2500 \text{ ft/sec}$)	> 50	$> 100 \text{ kPa}$ ($> 2000 \text{ psf}$)
D	$180 \text{ to } 360 \text{ m/sec}$ ($600 \text{ to } 1200 \text{ ft/sec}$)	$15 \text{ to } 50$	$50 \text{ to } 100 \text{ kPa}$ ($1000 \text{ to } 2000 \text{ psf}$)
E	$< 180 \text{ m/sec}$ ($< 600 \text{ ft/sec}$)	$< 15 \text{ blows/0.30 m}$ (15 blows/ft)	$< 50 \text{ kPa}$ ($< 1000 \text{ psf}$)

NOTE: If the \bar{S}_u method is used and the \bar{N}_{ch} and \bar{S}_u criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).

F Soils requiring site-specific evaluations:

1. Peats and/or highly organic clays ($H > 3$ m [10 ft] of peat and/or highly organic clay where H = thickness of soil)
2. Very high plasticity clays ($H > 8$ m [25 ft] with $PI > 75$)
3. Very thick soft/medium stiff clays ($H > 36$ m [120 ft])

When the soil properties are not known in sufficient detail to determine the Site Class, Site Class D may be used. Consequently Site Classes E or F need not be assumed unless the authority having jurisdiction determines that Site Classes E or F could be present at the site or in the event that Site Classes E or F are established by geotechnical data.

The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated on the basis of shear wave velocities in similar competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

The hard rock, Site Class A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 30 m (100 ft) surficial shear wave velocity measurements may be extrapolated to assess \bar{v}_s .

The rock categories, *Site Classes A and B*, shall not be used if there is more than 3 m (10 ft) of soil between the rock surface and the bottom of the spread footing or mat foundation.

3.4.2.2 Definitions of Site Class Parameters

The definitions presented below apply to the upper 30 m (100 ft) of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 30 m (100 ft). The subscript i then refers to any one of the layers between 1 and n .

The average \bar{v}_s for the layer is as follows:

$$\bar{v}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}} \quad (3.4.2.2-1)$$

where $\sum_{i=1}^n d_i$ is equal to 30 m (100 ft), v_{si} is the shear wave velocity in m/s (ft/sec) of the layer, and d_i is the thickness of any layer between 0 and 30 m (100 ft).

N_i is the Standard Penetration Resistance (ASTM D1586-84) not to exceed 100 blows/0.30 m (100 blows/ft) as directly measured in the field without corrections.

\bar{N} is:

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (3.4.2.2-2)$$

\bar{N}_{ch} is

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^m \frac{d_i}{N_i}} \quad (3.4.2.2-3)$$

where $\sum_{i=1}^m d_i = d_s$

In Equation 3.4.2.2-3, d_i and N_i are for cohesionless soils only and d_s is the total thickness of cohesionless soil layers in the top 30 m (100 ft).

s_{ul} is the undrained shear strength in kPa (psf), not to exceed 250 kPa (5,000 psf), ASTM D2166-91 or D2850-87.

\bar{s}_u is:

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^k \frac{d_i}{s_{ui}}} \quad (3.4.2.2-4)$$

w is the moisture content in percent, ASTM D2216-92.

3.4.2.3 Site Coefficients

where $\sum_{i=1}^k d_i = d_c$

d_c is the total thickness [(30- a_s) m (100- a_s) ft] of cohesive soil layers in the top 30 m (100 ft).

PI is the plasticity index, ASTM D4318-93.

Site coefficients for the short-period range (F_a) and for the long-period range (F_v) are given in Tables 3.4.2.3-1 and 3.4.2.3-2, respectively. Application of these coefficients to determine elastic seismic response coefficients of ground motions is described in Article 3.4.1

Table 3.4.2.3-1 Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at Short Periods				
	$S_s \leq 0.25 \text{ g}$	$S_s = 0.50 \text{ g}$	$S_s = 0.75 \text{ g}$	$S_s = 1.00 \text{ g}$	$S_s \geq 1.25 \text{ g}$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	<i>a</i>	<i>a</i>	<i>a</i>	<i>a</i>	<i>a</i>

NOTE: Use straight line interpolation for intermediate values of S_s , where S_s is the spectral acceleration at 0.2 seconds obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3). For the purpose of defining Seismic Hazard Levels in Article 3.7 Type E values may be used for Type F soils.

Table 3.4.2.3-2 Values of F_v as a Function of Site Class and Mapped 1 Second Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at 1 Second Periods				
	$S_1 \leq 0.1 \text{ g}$	$S_1 = 0.2 \text{ g}$	$S_1 = 0.3 \text{ g}$	$S_1 = 0.4 \text{ g}$	$S_1 \geq 0.5 \text{ g}$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	<i>a</i>	<i>a</i>	<i>a</i>	<i>a</i>	<i>a</i>

NOTE: Use straight line interpolation for intermediate values of S_1 , where S_1 is the spectral acceleration at 1.0 second obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3). For the purpose of defining Seismic Hazard Levels in Article 3.7 Type E values may be used for Type F soils.

3.4.3 Response Spectra Based on Site-Specific Procedure

A site-specific procedure to develop design response spectra of earthquake ground motions shall be performed when required by Article 3.4 and may be performed for any site. A site-specific probabilistic ground motion analysis shall include the following: characterization of seismic sources and ground motion attenuation that incorporates current scientific interpretations, including uncertainties in seismic source and ground motion models and parameter values; detailed documentation; and detailed peer review (Article C3.4.1).

Where analyses to determine site soil response effects are required by Articles 3.4 and 3.4.2.1 for Site Class F soils, the influence of the local soil conditions shall be determined based on site-specific geotechnical investigations and dynamic site response analyses.

For sites located within 10km (6 miles) of an active fault (as defined in Article 3.4), studies shall be considered to quantify near-fault effects on ground motions to determine if these could significantly influence the bridge response.

In cases where the 0.2-second or 1.0-second response spectral accelerations of the site-specific probabilistic response spectrum for the MCE exceeds the response spectrum shown in Figure 3.4.3-1, a deterministic spectrum may be utilized in regions having known active faults if the deterministic spectrum is lower than the probabilistic spectrum. The deterministic spectrum shall be the envelope of median-plus-one standard-deviation spectra calculated for characteristic maximum magnitude earthquakes on known active faults, but shall not be lower than the spectrum shown in Figure 3.4.3-1. If there is more than one active fault in the site region, the deterministic spectrum shall be calculated as the envelope of spectra for the different faults. Alternatively, deterministic spectra may be defined for each fault, and each spectrum, or the spectrum that governs bridge response, may be used for the analysis of the bridge.

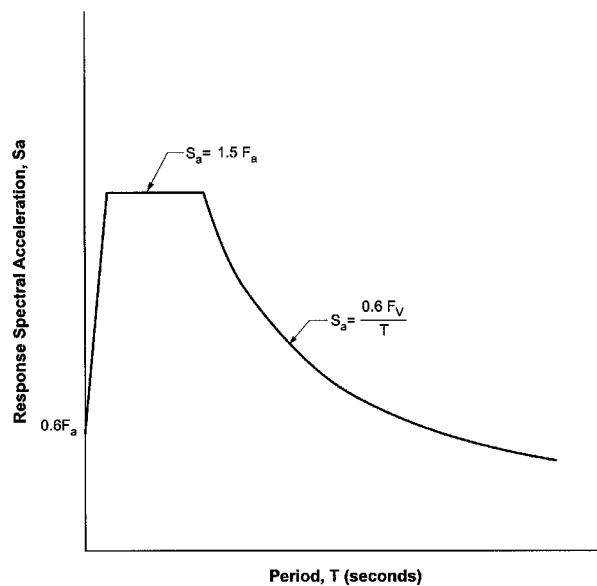


Figure 3.4.3-1 Minimum Deterministic Response Spectrum

When response spectra are determined from a site-specific study, the spectra shall not be lower than two-thirds of the response spectra determined using the general procedure in Article 3.4.1.

3.4.4 Acceleration Time Histories

The development of time histories shall meet the requirements of this section. The developed time histories shall have characteristics that are representative of the seismic environment of the site and the local site conditions.

Time histories may be either recorded time histories or spectrum-matched time histories. If sufficient recorded motions are not available, simulated-recorded time histories may be developed using theoretical ground motion modeling methods that simulate the earthquake rupture and the source-to-site seismic wave propagation.

If spectrum-matched time histories are developed, the initial time histories to be spectrum matched shall be representative recorded or simulated-recorded motions. Analytical techniques used for spectrum matching shall be demonstrated to be capable of achieving seismologically realistic time series that are similar to the time series of the initial time histories selected for spectrum matching.

When using recorded or simulated-recorded time histories, they shall be scaled to the approximate level of the design response spectrum in the period range of significance. For each component of motion, an aggregate match of the design response spectrum shall be achieved for the set of acceleration time histories used. A mean spectrum of the individual spectra of the time histories shall be calculated period-by-period. Over the defined period range of significance, the mean spectrum shall not be more than 15% lower than the design spectrum at any period, and the average of the ratios of the mean spectrum to the design spectrum shall be equal to or greater than unity. When developing spectrum-matched time histories, before the matching process, they shall be scaled to the approximate level of the design response spectrum in the period range of significance. Thereafter, the set of time histories for each component shall be spectrum-matched to achieve the aggregate fit requirement stated above.

At least three time histories shall be used for each component of motion for use in nonlinear inelastic time history analysis using either recorded, simulated-recorded, or spectrum-matched motions for either the 3% PE in 75 yr/1.5 mean deterministic or 50% PE in 75 yr event. The design actions shall be taken as the maximum response calculated for the three ground motions in each principal direction. If a minimum of seven time histories are used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction.

3.4.5 Vertical Acceleration Effects

The impact of vertical ground motion may be ignored if the bridge site is greater than 50km (32 miles) from an active fault as defined in Article 3.4 and may be ignored for all bridges in the central and Eastern U.S. as well as those areas impacted by subduction earthquakes in the Pacific Northwest. If the bridge site is located within

10km (6 miles) of an active fault then a site specific study is required if the response of the bridge could be significantly and adversely affected by vertical ground motion characteristics. In such cases response spectra and acceleration time histories as appropriate shall be developed and shall include appropriate vertical ground motions for inclusion in the design and analysis of the bridge. For vertical design forces the linear analysis shall use the CQC modal combination method and the SRSS directional combination method.

If the bridge site is located between 10km (6 miles) and 50km (32 miles) of an active fault a site specific study may be performed including the effects of appropriate vertical ground motion.

In lieu of a dynamic analysis that incorporates the effect of vertical ground motions, the following variations in column axial loads and superstructure moments and shears shall be included in the design of the columns and the superstructure to account for the effects of vertical ground motion.

$$\text{Column Axial Loads (AL)} = \text{DL Axial Force} \pm C_v (\text{DL Axial Force})$$

$$\text{Superstructure Bending Moments} = \text{DL Moment} \pm C_v (\text{DL Moment})$$

$$\text{Superstructure Shears} = \text{DL Shear} \pm C_v (\text{DL Shear})$$

C_v is the coefficient given in Table 3.4.5-1 if the maximum magnitude of the design earthquake is 7.0 or less, or Table 3.4.5-2 if the maximum magnitude of the design earthquake is greater than 7.0. Note that the coefficient C_v for the superstructure has a value specified at the mid-span location and at the column/pier support. Linear interpolation is used to determine C_v for points on the superstructure between these locations.

Table 3.4.5-1 Fault distance zones and corresponding dead load multiplier (C_v) for all bridges for rock and soil site conditions and a magnitude 7.0 event or less.

Response Quantity	Fault Distance Zones (km)				
	0-10	10-20	20-30	30-40	40-50
Pier Axial Force DL Multiplier					
	0.7	0.3	0.20	0.1	0.1
Superstructure Shear Force at Pier DL Multiplier					
	0.7	0.4	0.2	0.1	0.1
Superstructure Bending Moment at Pier DL Multiplier					
	0.6	0.3	0.2	0.1	0.1
Superstructure Shear Force at Mid-Span DL Multiplier					
	0.1	0.1	0.1	0.1	0.1
Superstructure Bending Moment at Mid-Span DL Multiplier					
	1.4	0.7	0.4	0.3	0.2
Footnotes (1) The DL Multiplier values given above are in addition to the dead load; thus, an actual "load factor" would be 1.0 plus/minus the above numbers. (2) The Live Load (LL) typically used in the design of bridge types shown in this study is in the range of 20-30% of the Dead Load (DL).					

Table 3.4.5-2 Fault distance zones and corresponding dead load multiplier (C_v) for all bridges for rock and soil site conditions and an event magnitude greater than 7.0.

Response Quantity	Fault Distance Zones (km)				
	0-10	10-20	20-30	30-40	40-50
Pier Axial Force DL Multiplier					
	0.9	0.4	0.2	0.2	0.1
Superstructure Shear Force at Pier DL Multiplier					
	1.0	0.5	0.3	0.2	0.2
Superstructure Bending Moment at Pier DL Multiplier					
	1.0	0.5	0.3	0.2	0.2
Superstructure Shear Force at Mid-Span DL Multiplier					
	0.2	0.1	0.1	0.1	0.1
Superstructure Bending Moment at Mid-Span DL Multiplier					
	1.9	1.0	0.6	0.5	0.3
Footnotes (1) The DL Multiplier values given above are in addition to the dead load; thus, an actual “load factor” would be 1.0 plus/minus the above numbers. (2) The Live Load (LL) typically used in the design of bridge types shown in this study is in the range of 20-30% of the Dead Load (DL).					

3.5 LOAD FACTORS

EXTREME EVENT-I (Table 3.5-1) – Load combination including maximum considered and expected earthquakes.

The load factor for live load in Extreme Event Load Combination I, γ_{EQ} , shall be determined on a project specific basis. The inertia effects of live

load do not need to be considered when performing a dynamic analysis. It is generally not necessary to consider the gravity effects of live load for Extreme Event-I except for bridges with heavy truck traffic (i.e. high ADTT) and/or elements particularly sensitive to gravity loading such as C-bents, outrigger bents or superstructures with nonsymmetrical geometry.

Table 3.5-1 Load Combinations and Load Factors

Load Combination Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH-I (unless noted)	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-IV EH, EV, ES, DW DC ONLY	γ_p 1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
STRENGTH-V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME EVENT-I	1.00	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
EXTREME EVENT-II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
SERVICE-III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
FATIGUE-LL, IM & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-	-

3.6 COMBINATION OF SEISMIC FORCE EFFECTS

The maximum seismic force due to seismic load in any one direction shall be based on the CQC combination of modal responses due to ground motion in that direction. The maximum force due to two or three orthogonal ground motion components shall be obtained either by the SRSS combination or the 100% - 40% combination forces due to the individual seismic loads.

3.6.1 SRSS Combination Rule

The maximum response quantity of interest is the SRSS combination of the response quantity from each of the orthogonal directions. (i.e., $M_x = \sqrt{(M_x^T)^2 + (M_x^L)^2}$ where M_x^T and M_x^L are the x-component moments from a transverse and longitudinal analysis)

If biaxial design of an element is important (e.g. circular columns) and the bridge has a maximum skew angle less than 10 degrees and/or a subtended angle less than 10 degrees then the maximum response quantities in the two

orthogonal directions (M_x , M_y) may use the 100% - 40% rule prior to obtaining the vector sum. The maximum vector moment is the maximum of:

$$\sqrt{M_x^2 + (0.4M_y)^2} \text{ or } \sqrt{(0.4M_x)^2 + M_y^2} \quad (3.6-1)$$

If the maximum skew angle or the subtended angle in a horizontally curved bridge exceeds 10 degrees then the maximum response quantities in the two horizontal directions shall be combined as the vector sum:

$$\sqrt{M_x^2 + M_y^2} \quad (3.6-2)$$

3.6.2 100% - 40% Combination Rule

The maximum response quantity of interest shall be obtained from the maximum of two load cases.

Load Case 1 (LC1) – 100% of the absolute value of the response quantity resulting from the analysis in one orthogonal direction (transverse) added to 40% of the response quantity resulting from the analyses in the other orthogonal direction(s) (longitudinal).

$$M_x^{LC1} = 1.0M_x^T + 0.4M_x^L \quad (3.6-3)$$

Load Case 2 (LC2) – 100% of the absolute value of the response quantity resulting from an analysis in the other orthogonal direction (longitudinal) added to 40% of the response quantity resulting from an analysis in the original direction (transverse).

$$M_x^{LC2} = 0.4M_x^T + 1.0M_x^L \quad (3.6-4)$$

If biaxial design of an element is important then the maximum response quantities in the two orthogonal directions from each load case shall be combined to obtain a vectorial sum and the maximum vector from the two load cases shall be used for design, i.e., the maximum of:

$$\sqrt{(M_x^{LC1})^2 + (M_y^{LC1})^2} \text{ or } \sqrt{(M_x^{LC2})^2 + (M_y^{LC2})^2} \quad (3.6-5)$$

3.7 SEISMIC HAZARD LEVEL (SHL), SEISMIC DESIGN AND ANALYSIS PROCEDURE (SDAP) AND SEISMIC DESIGN REQUIREMENT (SDR)

Each bridge shall be assigned a Seismic Hazard Level that shall be the highest level determined by the value of $F_v S_1$ or $F_a S_s$ from Table 3.7-1 for the MCE event.

Table 3.7-1 Seismic Hazard Levels

Seismic Hazard Level	Value of $F_v S_1$	Value of $F_a S_s$
I	$F_v S_1 \leq 0.15$	$F_a S_s \leq 0.15$
II	$0.15 < F_v S_1 \leq 0.25$	$0.15 < F_a S_s \leq 0.35$
III	$0.25 < F_v S_1 \leq 0.40$	$0.35 < F_a S_s \leq 0.60$
IV	$0.40 < F_v S_1$	$0.60 < F_a S_s$

Notes:

1. For the purposes of determining the Seismic Hazard Level for Site Class E Soils (Article 3.4.2.3) the value of F_v and F_a need not be taken larger than 2.4 and 1.6 respectively when S_1 is less than or equal to 0.10 and S_s is less than 0.25.
2. For the purposes of determining the Seismic Hazard Level for Site Class F Soils (Article 3.4.2.3) F_v and F_a values for Site Class E soils may be used with the adjustment described in Note 1 above.

Each bridge shall be designed, analyzed and detailed for seismic effects in accordance with Table 3.7-2. Seismic Design and Analysis Procedures (SDAP) are described in Section 4. Minimum seismic design requirements (SDR) for SDR 1 and 2, SDR 3 and SDR 4 are given in Sections 6, 7 and 8, respectively.

Table 3.7-2 Seismic Design and Analysis Procedures (SDAP) and Seismic Design Requirements (SDR)

Seismic Hazard Level	Life Safety	
	SDAP	SDR
I	A1	1
II	A2	2
III	B/C/D/E	3
IV	C/D/E	4

Notes:

1. SDAP B/C – The use of these two design/analysis procedures is governed by regularity requirements as defined in Articles 4.3.2 and 4.4.2 respectively.
2. SDAP D – The use of the uniform load method is only permitted for the life safety performance level and limits on its use are given in Article 5.4.2.1.
3. If abutments are required to deform inelastically and act as part of the ERS then only SDAP D or E can be used and the uniform load method (ULM) is not permitted.
4. If owners approval of an ERE is required (Article 3.3.1 – i.e. inelastic behavior that is not inspectable occurs in a substructure) then SDAP E must be used.
5. Seismic Design Requirements (SDR) 1 and 2 are given in Section 6, SDR 3 is given in Section 7, and SDR 4 is given in Section 8.

3.8 TEMPORARY AND STAGED CONSTRUCTION

Any bridge or partially constructed bridge that is expected to be temporary for more than five years shall be designed using the requirements for permanent structures and shall not use the provisions of this Article.

The requirement that an earthquake shall not cause collapse of all or part of a bridge, as stated in Article 3.2, shall apply to temporary bridges expected to carry traffic. The provisions also apply to those bridges that are constructed in stages and expected to carry traffic and/or pass over routes that carry traffic. The design ground

response spectra given in Article 3.4 may be reduced by a factor of not more than 2 in order to calculate the component elastic forces and displacements. Response spectra for construction sites that are close to active faults shall be the subject of special study. The response modification factors given in Article 4.7 may be increased by a factor of not more than 1.5 in order to calculate the design forces. This factor shall not be applied to connections as defined in Table 4.7-2.

The minimum seat width provisions of Article 7.3 or 8.3 shall apply to all temporary bridges and staged construction.

Figure 3.4.1-2(a) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES, OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

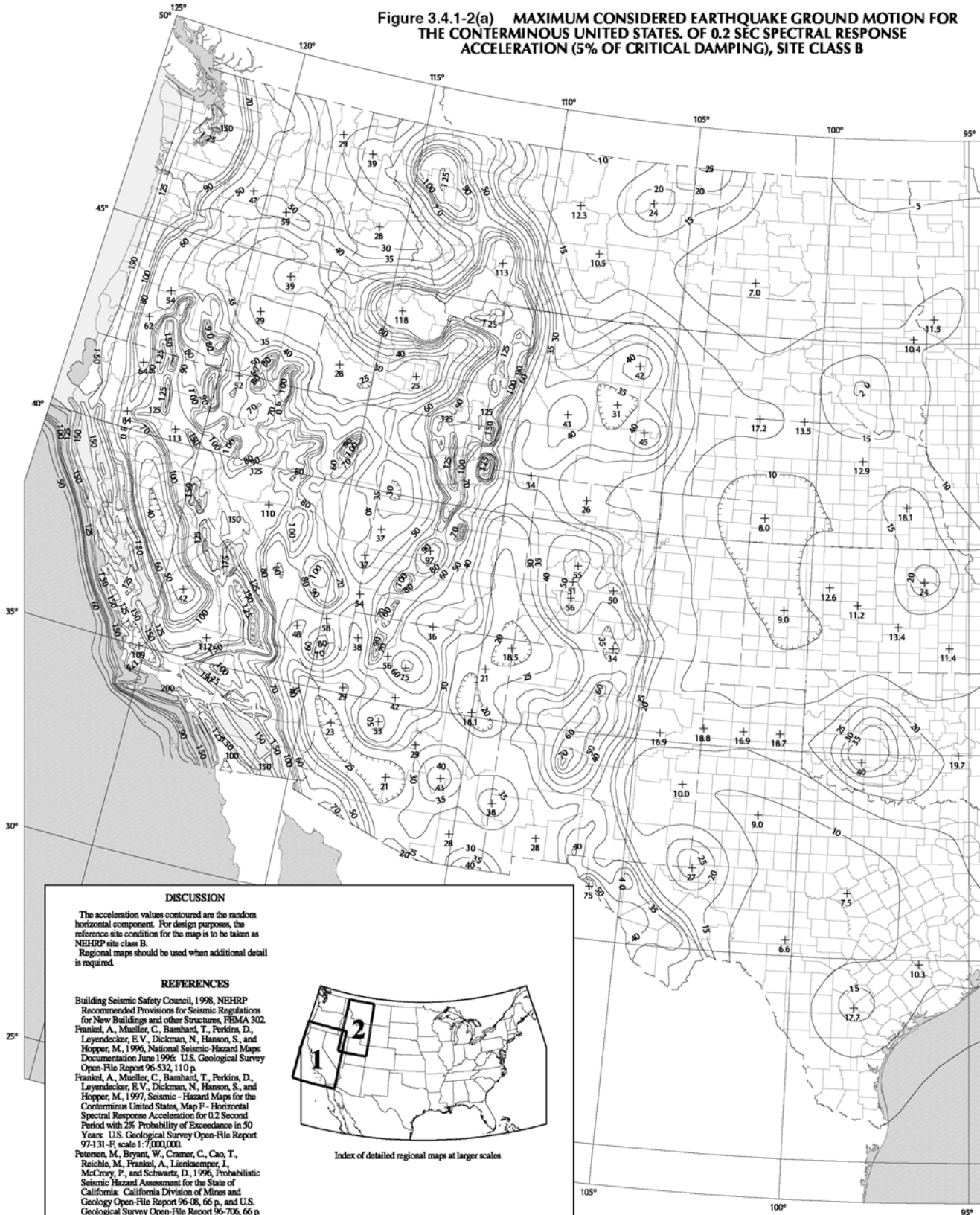


Figure 3.4.1-2(a) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES. OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

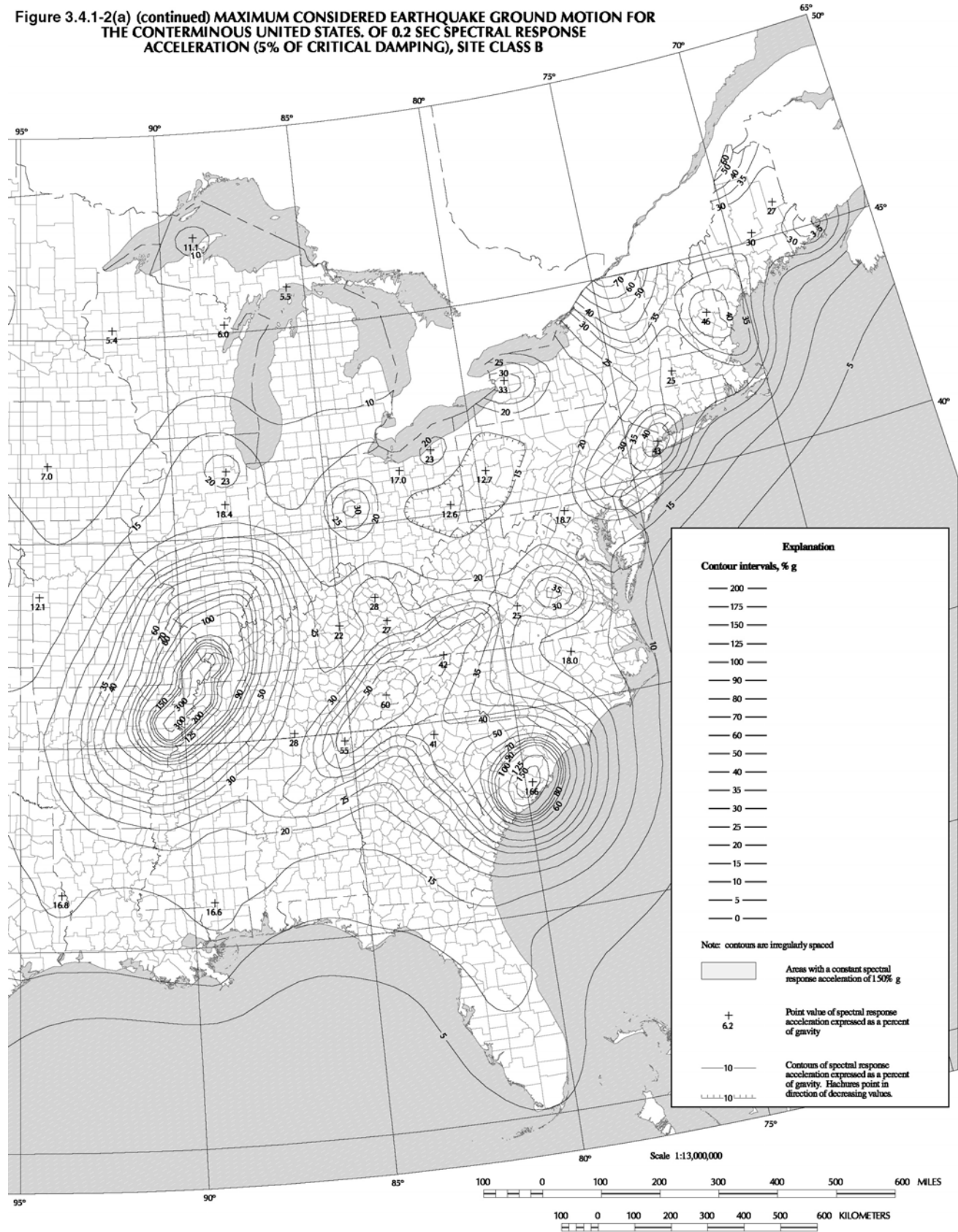


Figure 3.4.1-2(b) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES, OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

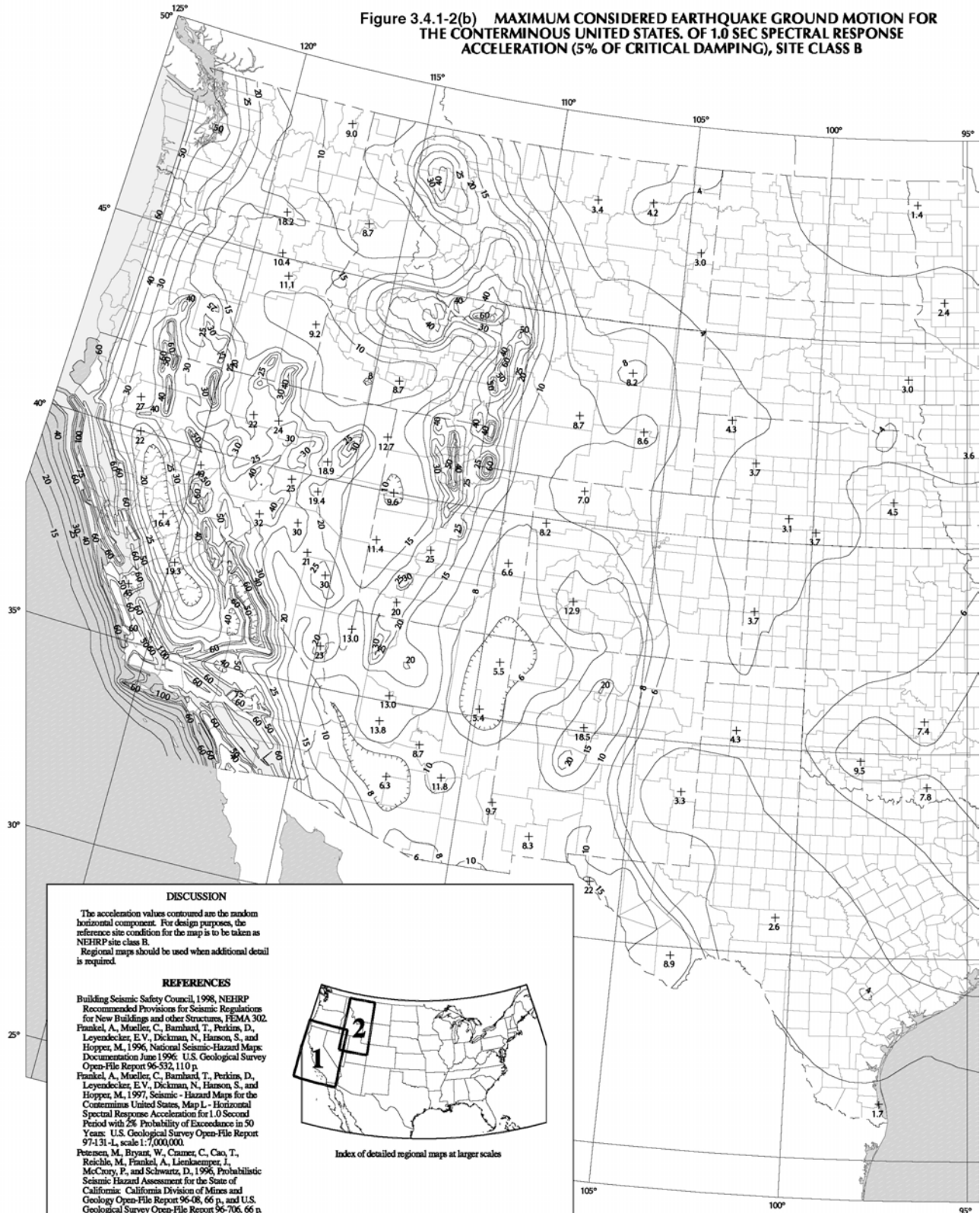


Figure 3.4.1-2(b)(continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES. OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

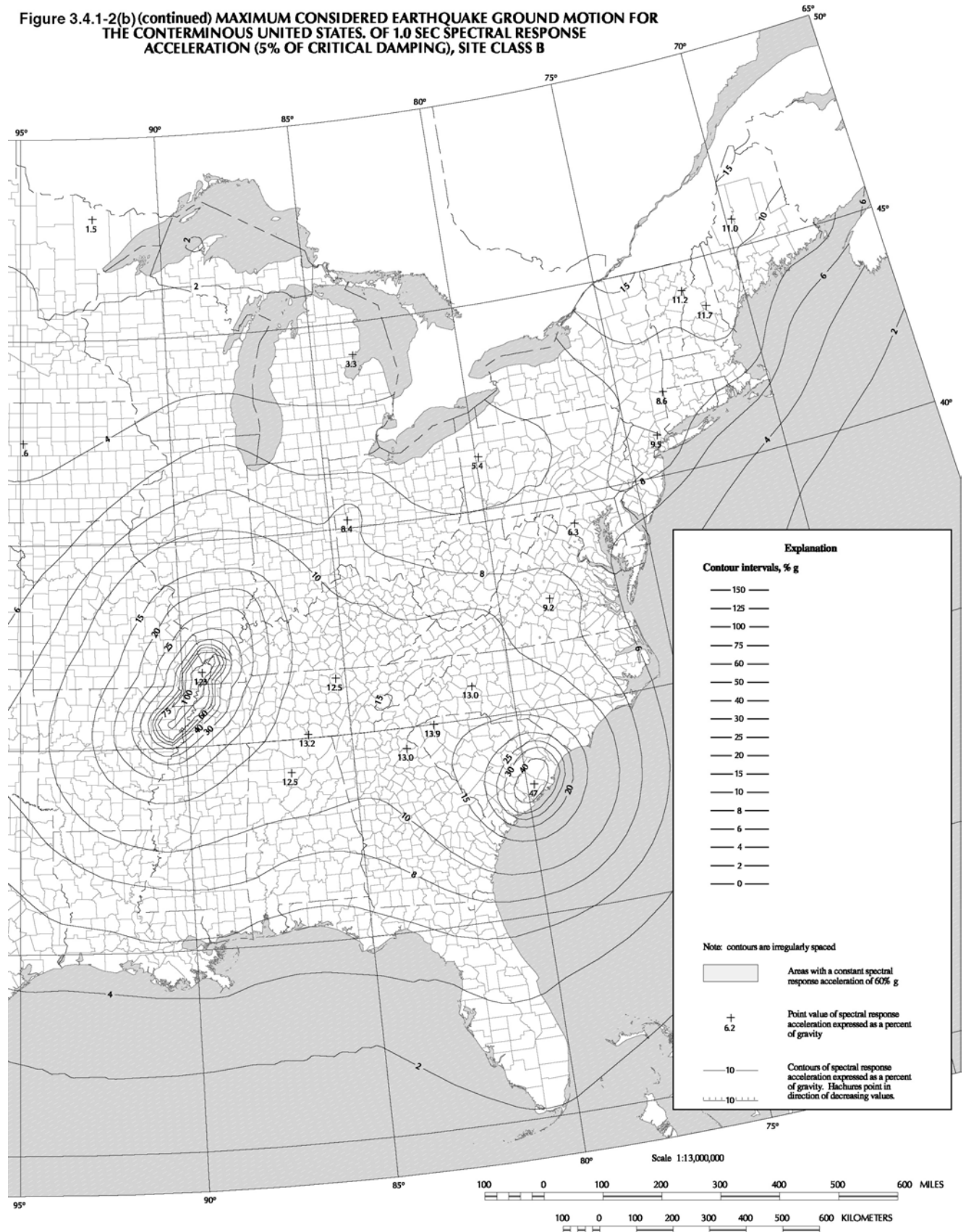
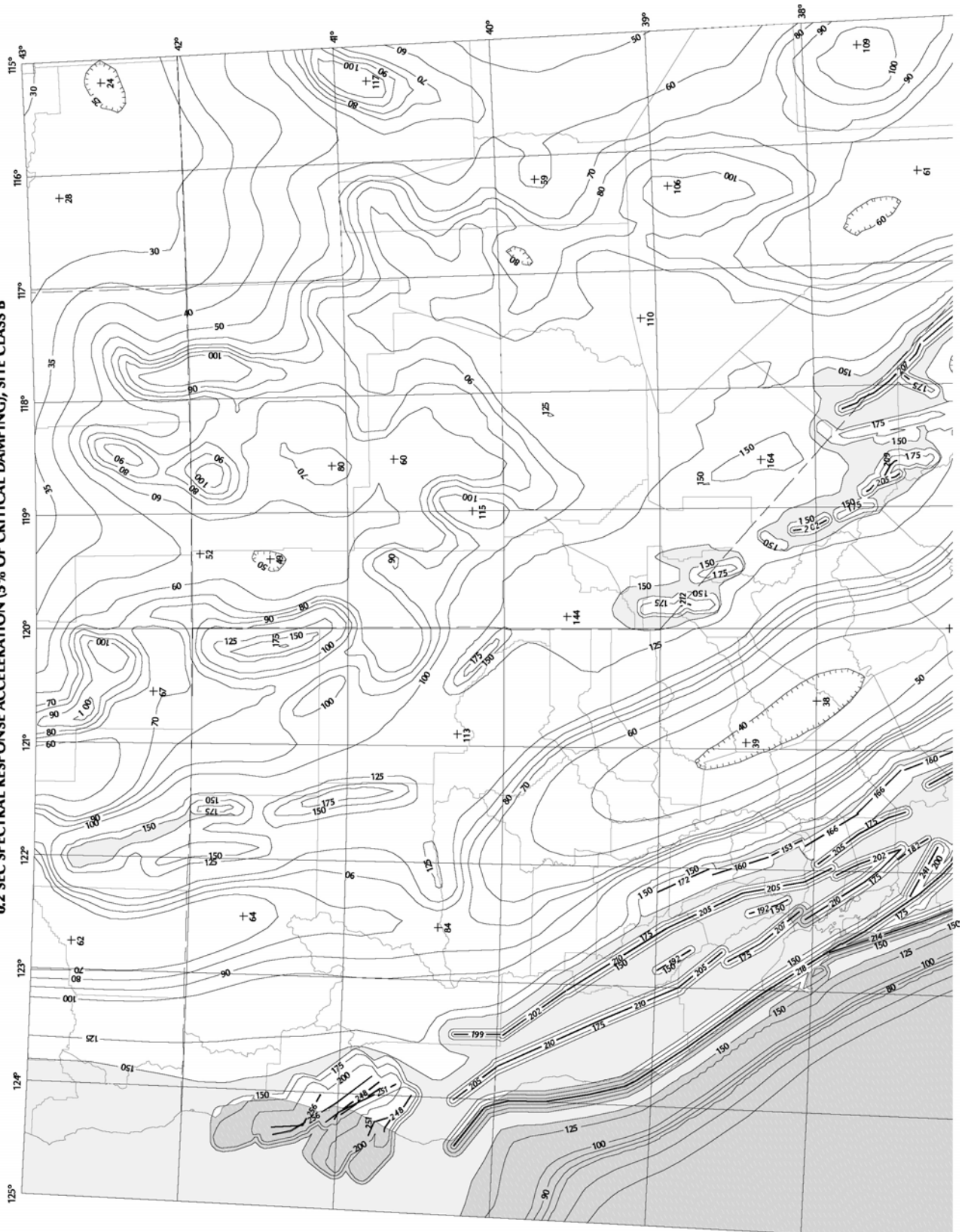


Figure 3.4.1-2(c) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



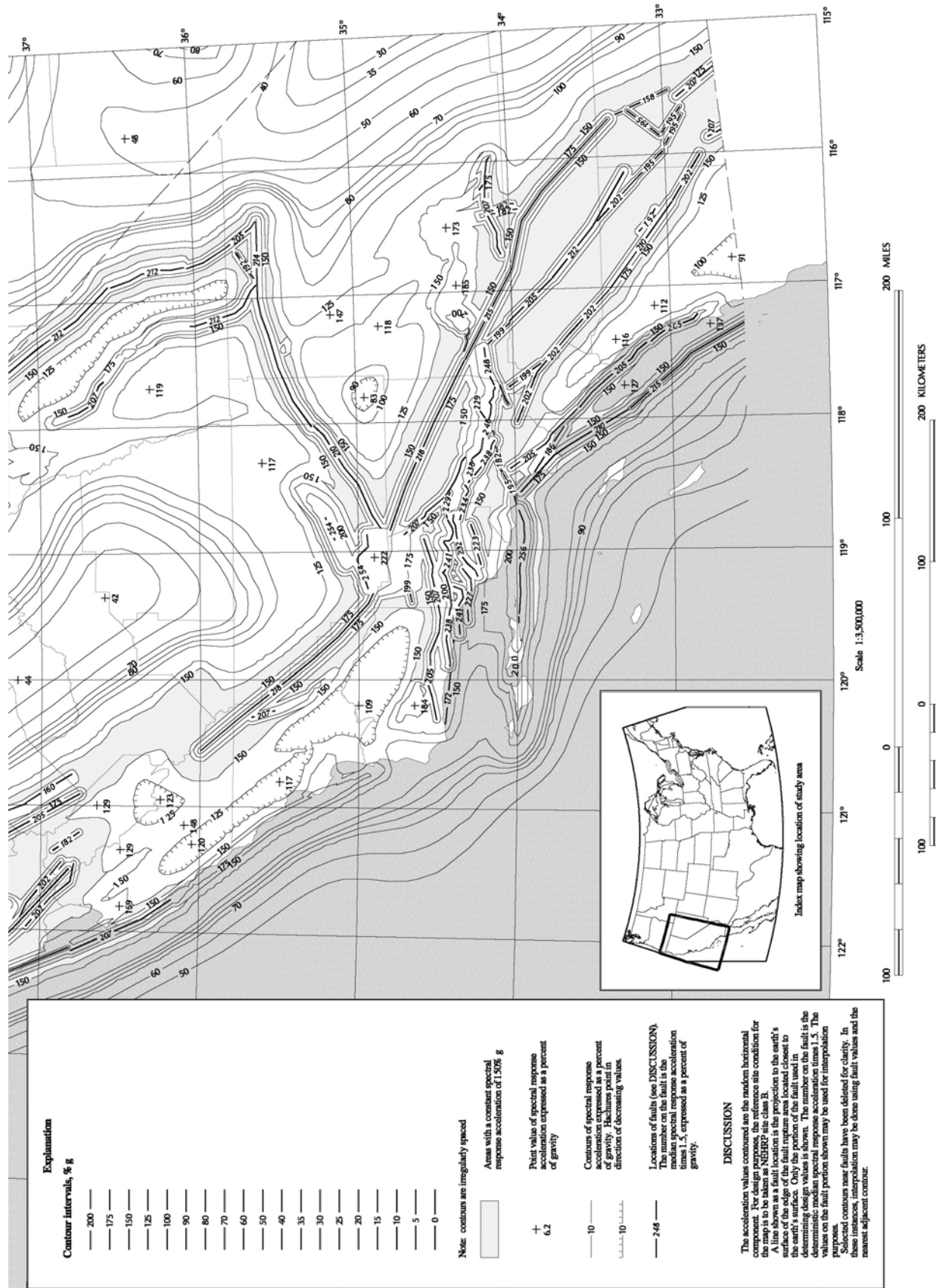


Figure 3.4.1-2(c) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

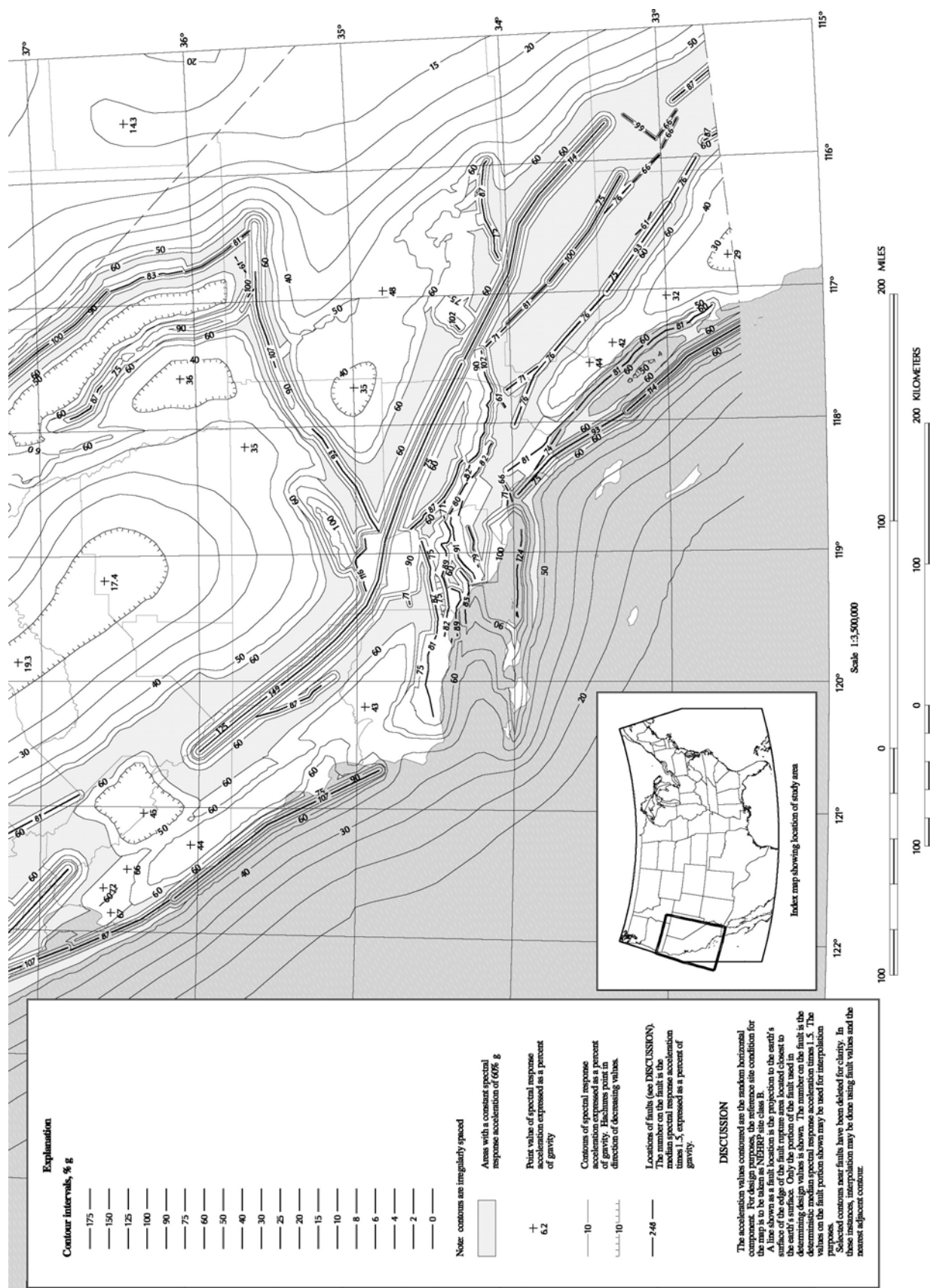


Figure 3.4.1-2(d) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

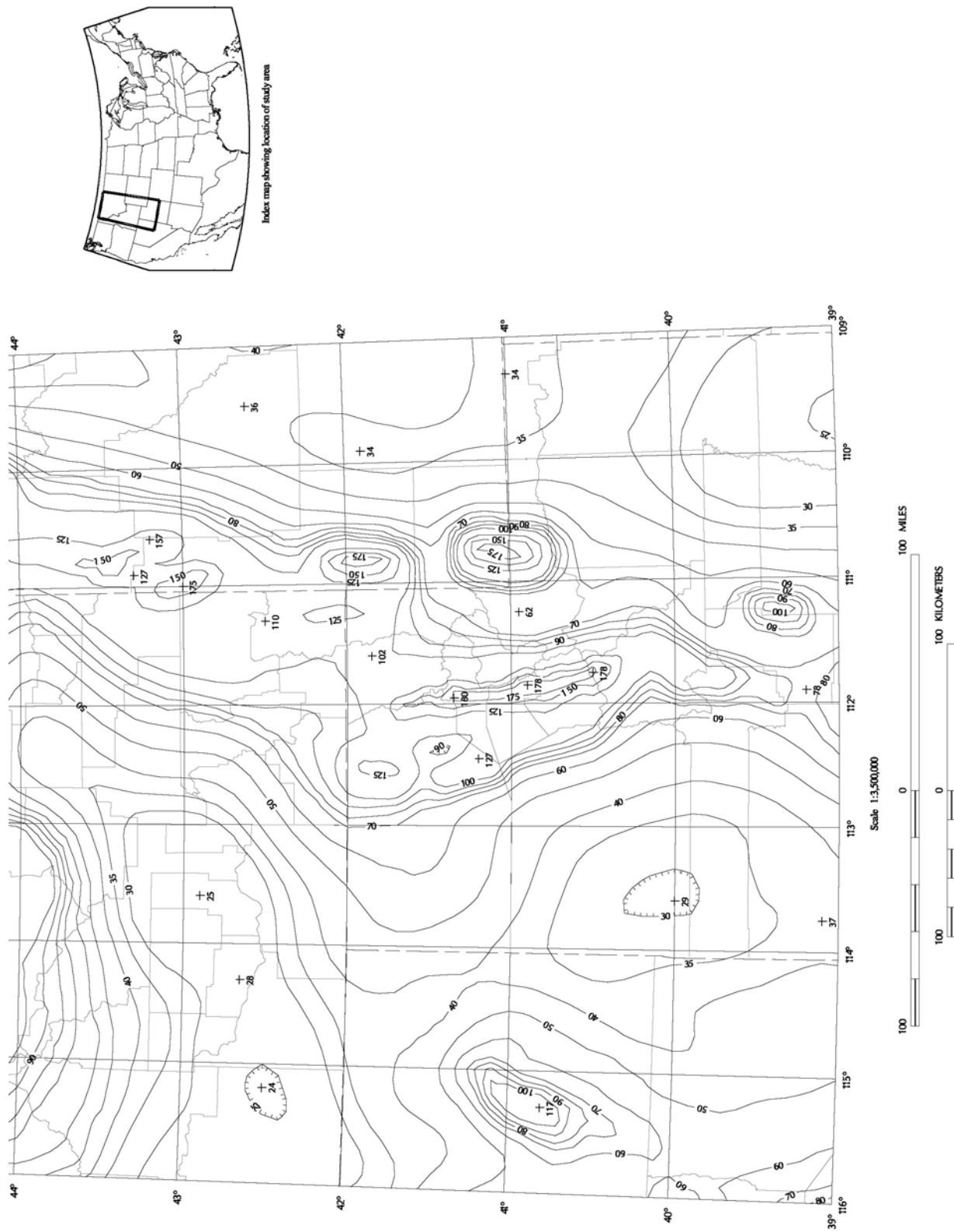
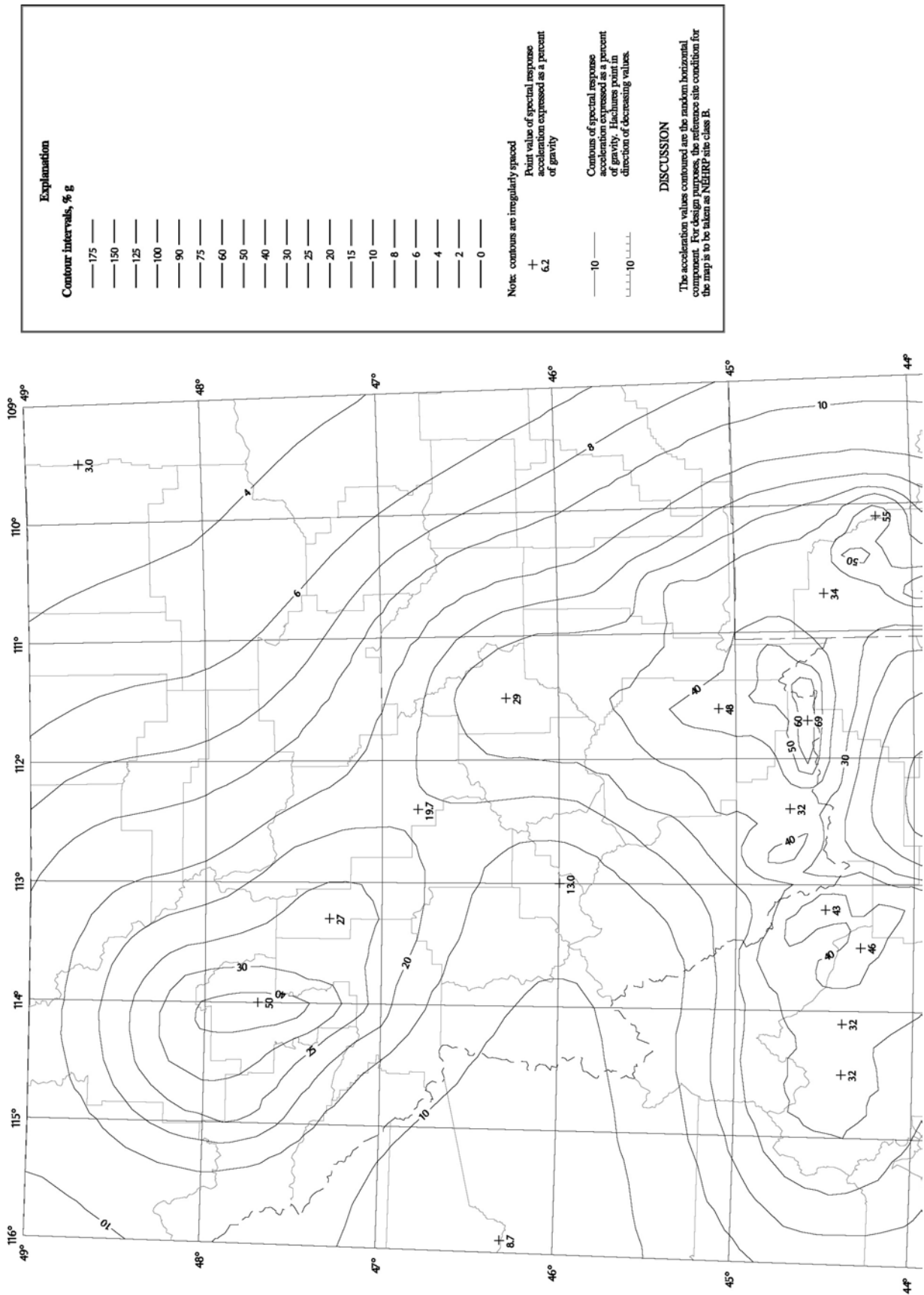


Figure 3.4.1-2(e) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Figure 3.4.1-2(f) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



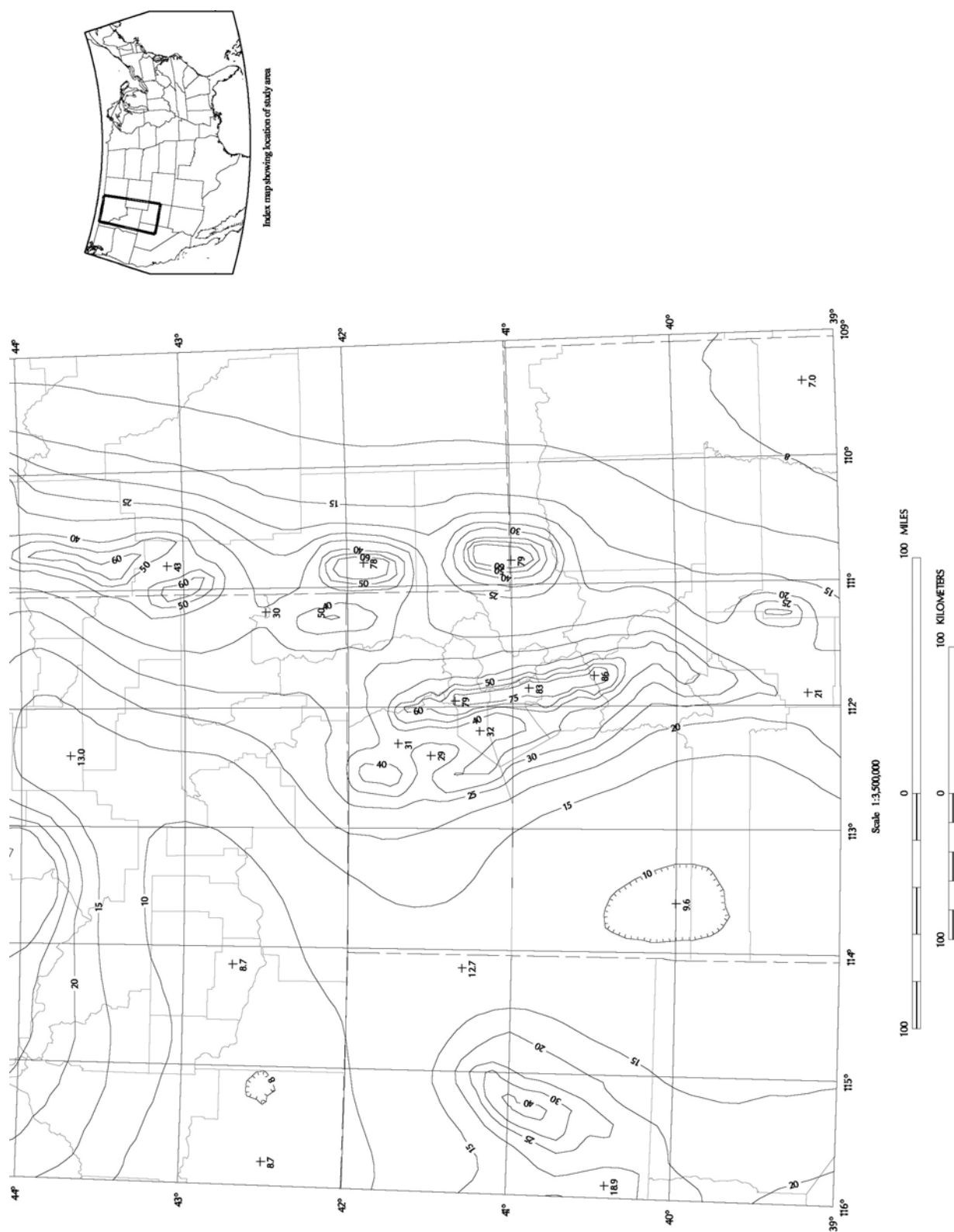


Figure 3.4.1-2(f) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Figure 3.4.1-2(g) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR ALASKA OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

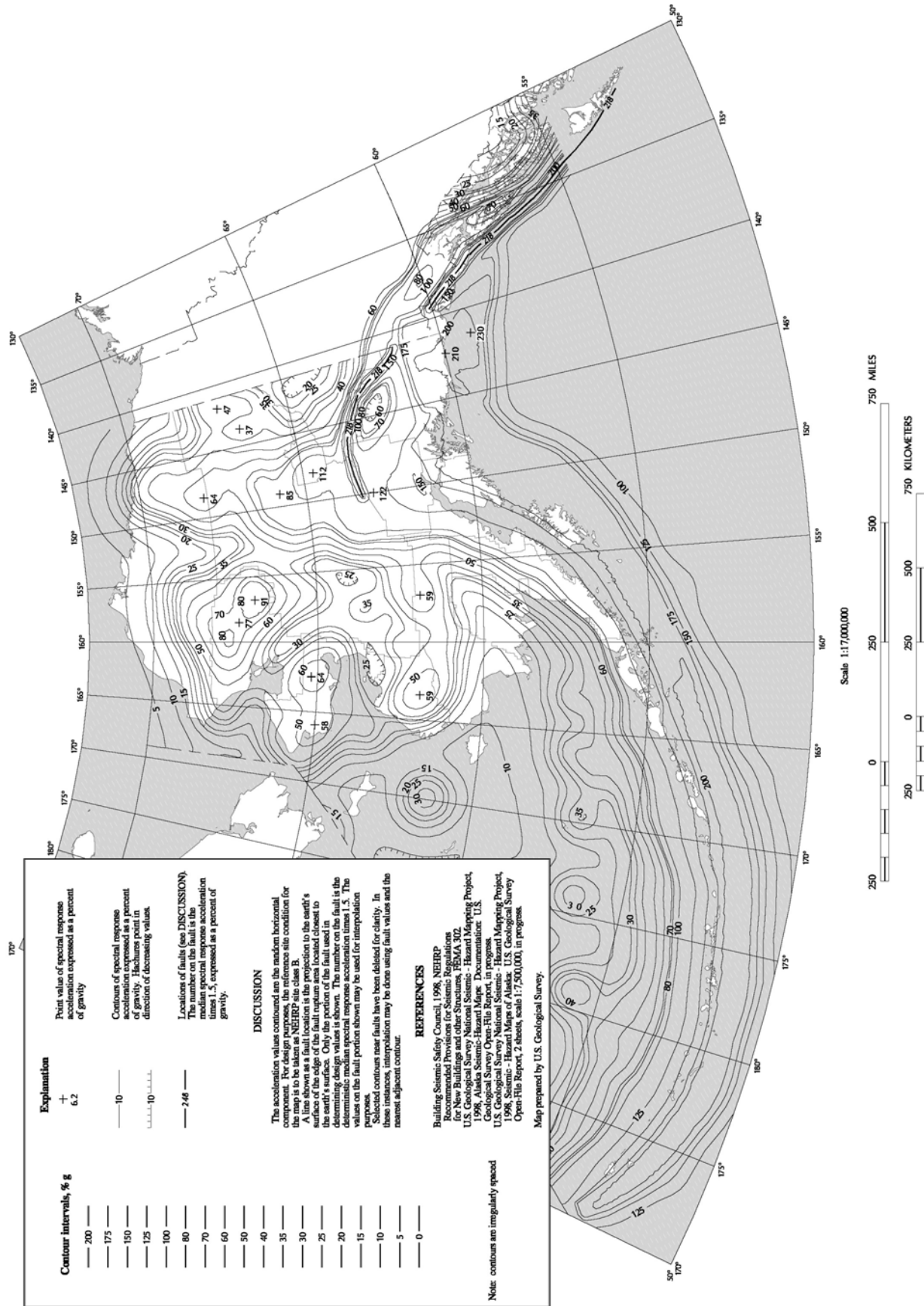


Figure 3.4.1-2(h) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR ALASKA OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

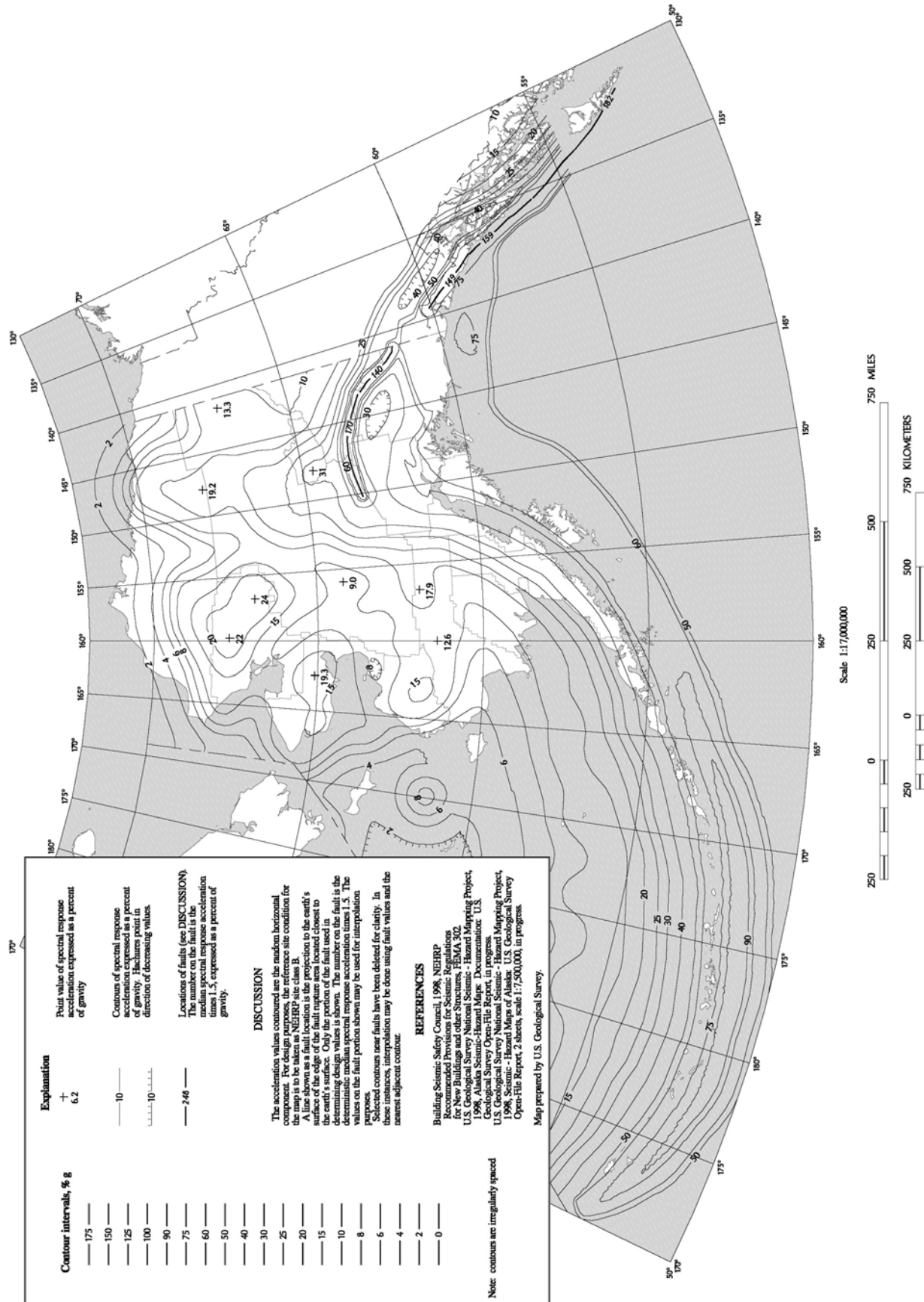


Figure 3.4.1-2(j) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR HAWAII OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

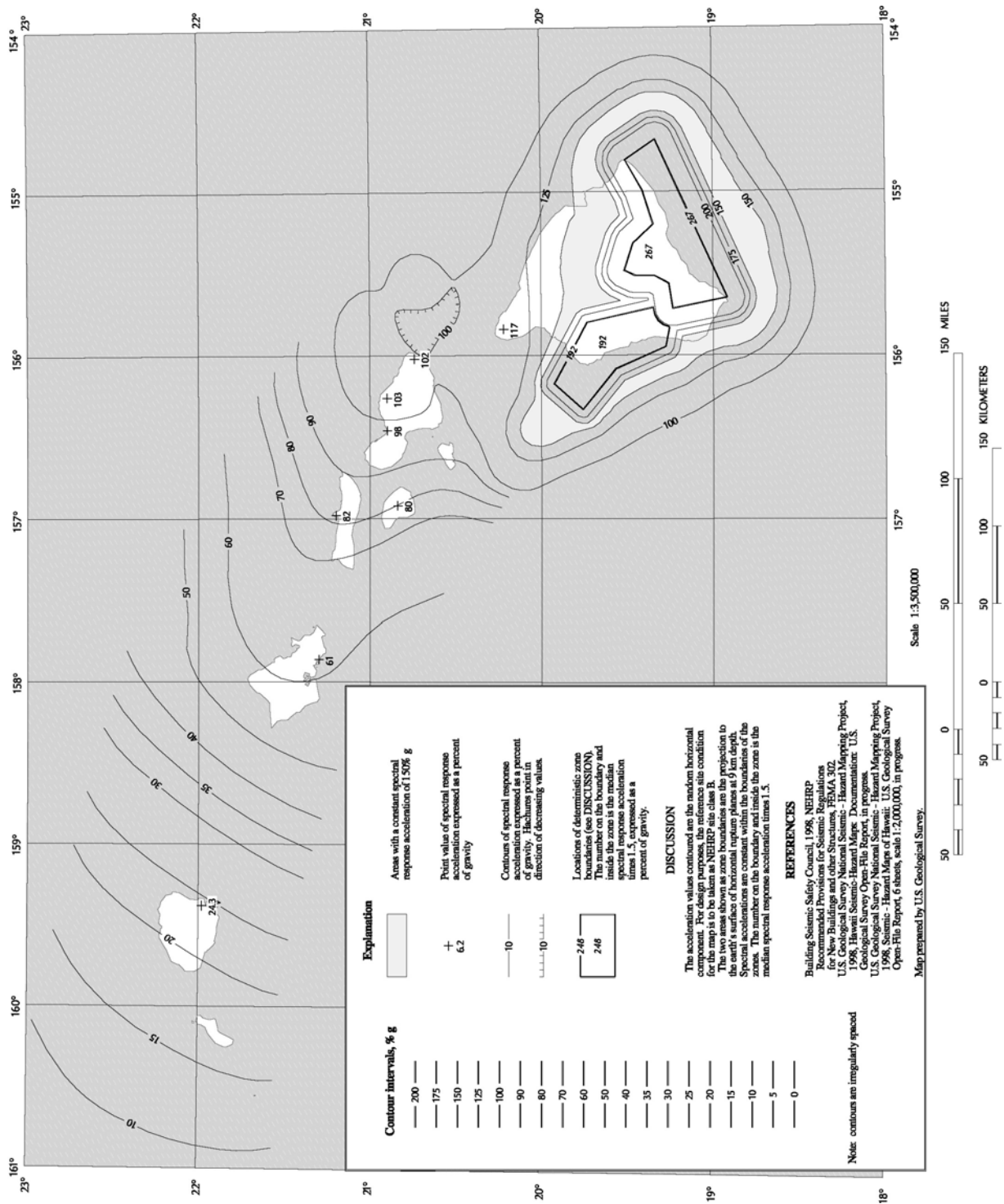


Figure 3.4.1-2(i) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR HAWAII OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

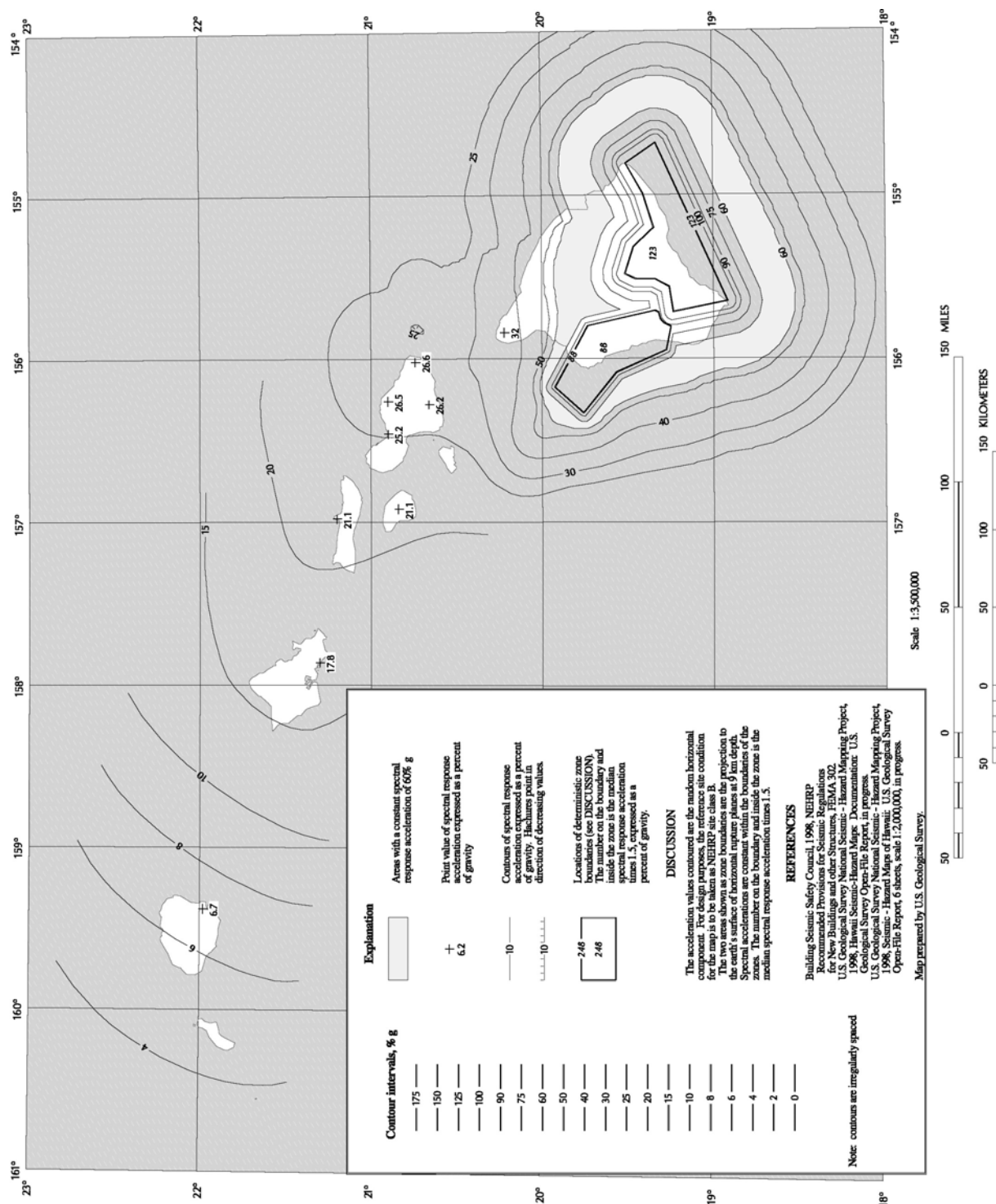


Figure 3.4.1-2(k) **MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR PUERTO RICO, CULEBRA, VIEQUES, ST. THOMAS, ST. JOHN, AND ST. CROIX OF 0.2 AND 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B**

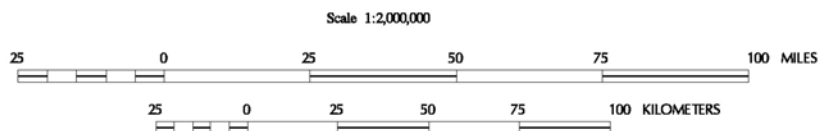
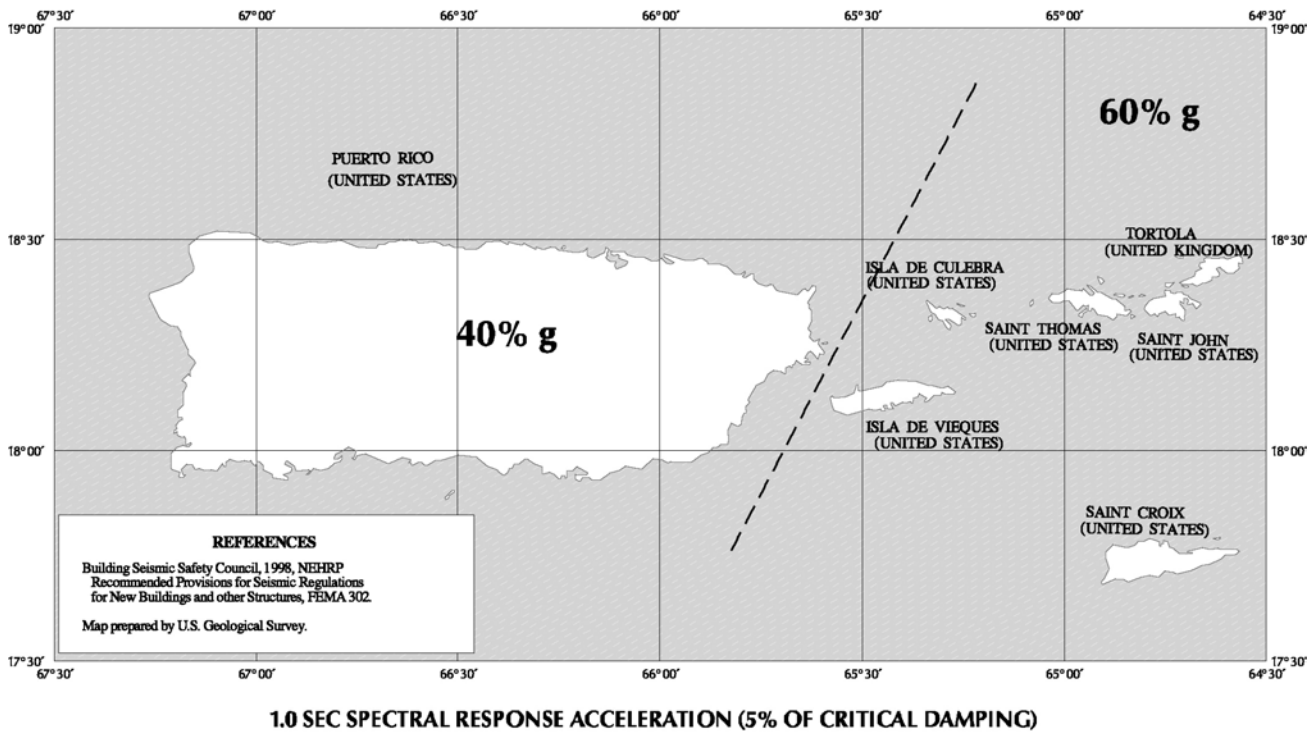
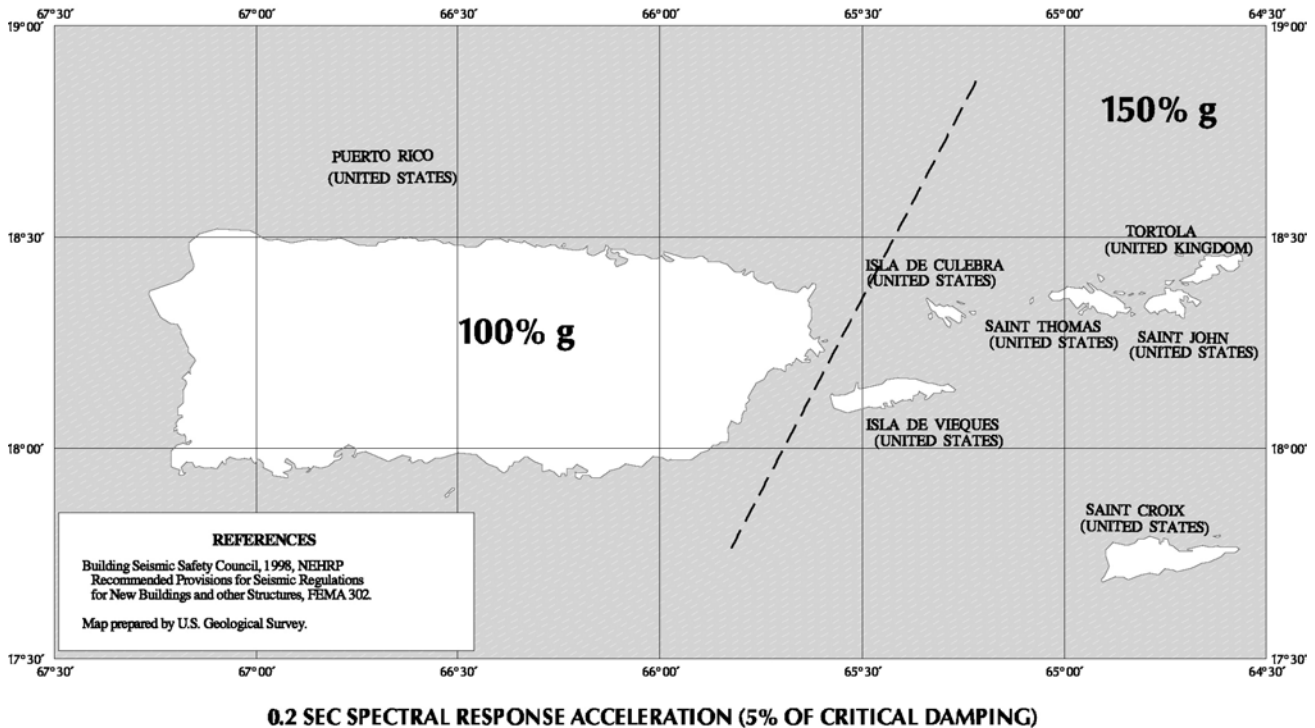


Figure 3.4.1-2(l) **MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR GUAM AND TUTUILA OF 0.2 AND 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B**

